Assessing the stability of piled tripod foundations for offshore wind turbines under cyclic loading

Bruno Stuyts[1], Jamie Irvine[2], David Cathie[1]
[1]Cathie Associates SA/NV, J.E. Mommaertslaan 22, B-1831 Diegem, Belgium
[2]Cathie Associates Ltd., Churchill House, 12 Mosley Street, NE 1 DE Newcastle Upon Tyne, U.K.
email: bruno.stuyts@cathie-associates.com, jamie.irvine@cathie-associates.com, david.cathie@cathie-associates.com

ABSTRACT: As offshore wind projects move further offshore into deeper waters, tripod and jacket structures are becoming the most economic method of supporting wind turbines. These support structures are subjected to cyclic loading arising from wind, waves and currents. Traditional offshore oil & gas tripods / jacket structures generally support large deadloads and the magnitude of the cyclic loading is low in comparison to these permanent loads. The wind turbine support structures are much lighter than traditional jackets meaning the cyclic loading is a significant proportion of the total loads. The cyclic loads are transferred to the foundation where they could cause degradation of the subsoil. The response of the piles to long term cyclic lateral and axial loads is complex and there are no generally accepted methodologies to predict the effect of cyclic loading on the foundation resistance. This paper describes the available methodologies to account for the potential effects of cyclic loading on the foundation resistance of tripod piles driven in dense sand. A rational design approach is outlined based on available field- or model scale results and past experience in the offshore oil & gas industry. The design approach shows how the effects of cyclic axial and lateral loading on pile resistance can be accounted for. The limitations of the design approach are reviewed as well as the inherent conservatism. Due to the relatively small database of pile tests, conservatism is inevitable. Further research into the long-term behavior of tripod piles is clearly warranted.

KEY WORDS: Foundation; Tripod; Cyclic; Wind; Pile; Offshore; Axial resistance; Lateral resistance.

1 INTRODUCTION

The response of tripod piles to long term cyclic lateral and axial loads is complex and there are no generally accepted methodologies to predict the effect of cyclic loading. Moreover, the majority of recent research on the effects of cyclic loading focused on monopiles. The response of monopiles is not directly applicable to the assessment of tripod or jacket piles due to the differing load regime.

For tripod piles, the cyclic axial loads will govern the design of the piles. Although the amplitudes of cyclic lateral loads are much lower than for monopiles, their effect also needs to be considered. The second section of this paper reviews the response of the soil adjacent to the pile to cyclic axial and lateral loading. A typical soil profile for the Southern North Sea consisting of dense to very dense sand is adopted for the analyses.

Cyclic lateral loading has an impact on axial pile resistance but cyclic axial loading does not significantly affect lateral response [1][2]. Therefore the two conditions can be uncoupled provided the effects of lateral loading treated first and the consequences carried forward to the axial analysis. In Section 3, the effect of cyclic loading on the lateral and axial resistance is assessed, taking the differences with the behavior of a monopile into account.

For tripod piles, the majority of the moments arising from wind and wave loading are transferred to axial loads on the individual piles. The degradation of axial capacity during storm events needs to be accounted for. The available methods for axial degradation analyses are reviewed in Section 4 together with the available pile test data.

In Section 5, a rational design approach is suggested based on the various calculation models from the literature. The limitations of this design approach are reviewed as well as the inherent conservatisms. Due to the relatively small database of pile tests, these conservatisms are inevitable. Further research into the long-term behavior of tripod piles is clearly warranted.

2 SOIL-PILE INTERACTION DUE TO CYCLIC LOADING

2.1 *Cyclic lateral loading*

Cyclic lateral loading can cause irreversible lateral displacements and therefore reduce the bending stiffness of the pile-soil system due to non-linear or plastic deformation of the soil surrounding the pile. Considering a constant intensity of cyclic lateral loading, the soil near the surface yields (Figure 1) and, as each load cycle takes place, the lateral load is transferred progressively down the pile. Finally, if the design is satisfactory, a shakedown condition is reached where no further degradation occurs and no further plastic work is dissipated at that loading intensity [3].

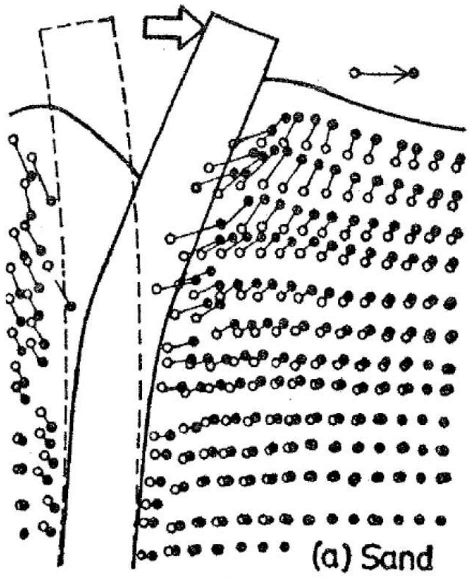

Figure 1. Movement of sand grains around a laterally loaded pile [4]

Long and Vanneste [5] reported 34 full-scale tests to identify the factors influencing the cyclic lateral behavior. Installation method, soil type, pile type and cyclic load characteristics were identified as having a significant effect on the response. The characteristics of the load was the most important contributor with two-way lateral loading causing significantly less degradation than one-way cyclic lateral loading.

Rosquet [6] also demonstrated the importance of the direction of loading based on lateral pile load tests in the centrifuge on dense to very dense Fontainebleau sand. The number of load cycles was limited to 15 for most tests. Large amplitude one-way loading can cause significant softening of the lateral response due to densification of the soil on the passive side of the pile. An improvement of lateral capacity was noticed during two-way loading. This is believed to be due additional sand grains filling the space between the pile and the soil on the active side. The sand on the passive side still densifies but the ingress of additional material prevents a softening of the pile-soil interaction.

For tripod piles, the loading regime is almost exclusively two-way loading due to the nature of the structure. Therefore, degradation of lateral capacity will not be as serious an issue for tripod piles as for monopiles.

Rakotonindriana [7] carried out centrifuge tests on single piles and pile groups of 2x2 piles under one-way loading. Tests were carried out for larger cycle numbers up to 75000. The author concluded that for cyclic one-way loading with maximum horizontal loads exceeding 10% of the ultimate horizontal load, degradation of the lateral stiffness could always be expected, regardless of the relative density of the sand. Two-way loading was not considered in his study.

LeBlanc et al [8] performed 1-g tests on rigid piles in sand and show the important differences between rigid and flexible behavior. A rigid pile rotates without flexing significantly and develops a “toe-kick” under moment and lateral loading. Flexible piles are fixed below a certain depth leading to elastic rebound of the pile upon unloading. The difference between rigid and flexible pile is illustrated in Figure 2.

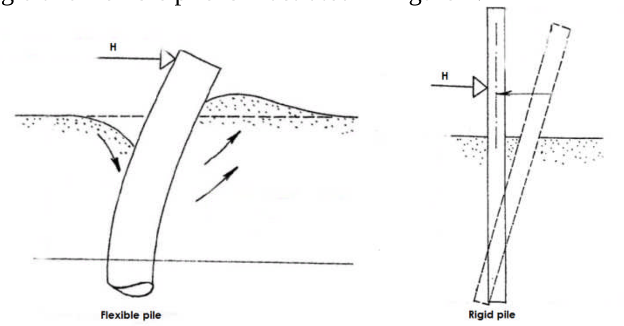

Figure 2. Flexible vs. rigid pile behavior under lateral load

Leblanc et al [8] propose a criterion to determine whether the pile behaves in a rigid or flexible manner:

$$4.8 < \frac{E_s L^4}{E_p I_p} < 388.6 \tag{1}$$

Where E_s is the elastic modulus of the soil, L is the pile length, E_P is the elastic modulus of the pile and I_p is the moment of inertia of the pile cross-section. The upper limit for rigid behavior is 4.8 and the lower limit for flexible behavior 388.6. In between these two limits, a transition zone exists between rigid and flexible behavior.

For typical tripod piles in dense to very dense sand, flexible behavior is always obtained. This makes the accumulation of large permanent displacements less likely.

Dührkop [9] also carried out 1-g model tests up to 10,000 cycles and focusing on one-way loading. The accumulation of pile head displacement is dependent on the direction and magnitude of the applied cyclic loads, the relative density of the soil and the number of applied load cycles.

Although the literature provides a relatively large test database of field scale, 1-g laboratory and centrifuge model tests, the majority of these tests are based on typical monopile dimensions and loading conditions. The main differences with tripod piles are:

- Tripod piles behave in a flexible rather than in a rigid manner.
- Large bending moments arising from wind and wave loading are redistributed to axial loads leading to much smaller bending moments acting at mudline compared to monopiles.
- The fixity at the pile head leads to a bending moment at mudline opposing the applied shear load (Figure 3).
- The majority of the loading is two-way cyclic lateral loading which has been shown to improve lateral capacity.

The combination of the four points given above leads to the conclusion that accumulation of lateral displacements is not likely to be a significant issue for piled tripods. However, the densification of the soil around the pile can lead to a reduction of the radial stress on the pile-soil interface. This will need to be considered in the calculation of the axial pile resistance.

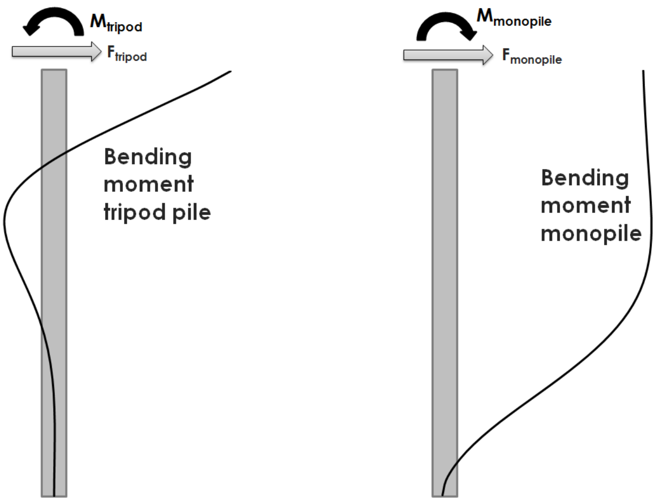

Figure 3. Bending moment comparison in tripod piles and monopiles

2.2 Cyclic axial loading

Cyclic axial loading in dense to very dense sand can cause degradation of the radial stress on the pile-soil interface as demonstrated in field tests carried out at the Dunkirk test site by Jardine et al [10]. The following mechanisms control the degradation of axial pile resistance:

- Cyclic axial loads with amplitudes larger than 25% of the shaft resistance cause progressive densification of the soil around the pile. Cycling below the threshold value can be beneficial for axial pile resistance.
- The densification of the soil leads to a reduced radial stress on the pile-soil interface.
- Reduction in the radial stress reduces the ultimate shaft friction leading to an increased utilization of the available unit shaft friction.
- Axial loads are transferred further down the pile as degradation increases.

To date, the test database from the Dunkirk test site remains the only available data source for full-scale cyclic axial pile tests in dense sand. Cycle numbers up to N=345 were tested.

Richter et al [11] present an extrapolation to higher cycle numbers based on small scale model tests. Although some authors do not make a distinction between cohesionless and cohesive soils, Richter et al [11] shows that such a distinction should be made due to the fundamentally different material behavior.

Accumulated axial pile head displacements are difficult to predict and depend on a number of factors:

- Soil type and soil stiffness.
- Magnitude of the applied cyclic axial load compared to the shaft resistance.
- Direction of the loads.
- Number of load cycles

Richter et al [11] propose a logarithmic equation to approximate the accumulated axial pile head displacements.

$$s_{cyc} = s_{N=1}(1 + t \ln N) \tag{2}$$

Where s_{cyc} is the accumulated displacement, $s_{N=1}$ is the axial displacement during the first load cycle, t is a parameter describing the system behavior and N is the number of cycles.

For piled tripods, the degradation of axial pile resistance and the accumulation of permanent axial displacements is much more important than the degradation of lateral resistance and the accumulation of permanent lateral displacements. Therefore, axial degradation should be considered carefully during the design.

3 INFLUENCE OF CYCLIC LATERAL LOADS ON LATERAL AND AXIAL PILE RESISTANCE

3.1 Influence on lateral response

Lateral response analysis is routinely carried out using P-Y methodology whereby the pile is supported by a series of elasto-plastic lateral springs. In silica sand, the ultimate resistance and the stiffness of the springs depends on the friction angle (and therefore the relative density) of the sand.

The P-Y construction proposed in API [12] is routinely used in offshore practice:

$$p(z) = A \cdot p_u(z) \tanh\left(\frac{k \cdot z}{A \cdot p_u(z)} y\right) \tag{3}$$

Where p is the lateral pressure in N/m, A is an empirical factor which takes a different form depending on whether static or cyclic loading is considered, y is the lateral deflection and k and p_u are the modulus of subgrade reaction and the ultimate lateral resistance of the soil. The last two properties depend on the friction angle of the sand.

Although the cyclic lateral resistance is reduced compared to the static lateral resistance through the use of the parameter A, the parameter was calibrated based on relatively small cycle numbers. Recent research ([7], [8], [9]) indicates that the P-Y response from API underestimates lateral deflections after large numbers of cycles. Therefore, modified P-Y constructions were suggested.

Long and Vanneste [5] propose a DSPY procedure (deterioration of static P-Y curve) where the ultimate lateral pressure and modulus of subgrade reaction are deteriorated from their initial static values depending on the nature of cyclic loading, pile installation method and sand relative density. Although the calculation model is successful in back-calculating the pile tests reported in [5], the model is not applicable to flexible pile since it deteriorates soil properties along the entire pile length. Recent model tests have shown that reductions of the P-Y response are limited to a certain depth below which no degradation occurs.

Rakotonindriana [7] also proposes a reduction of the static P-Y curves through the use of reduction coefficients (Figure 4). The reduction coefficients were determined from centrifuge tests on instrumented pile groups of 2x2 pile undergoing lateral loads. The proposed reduction coefficients depend on the relative density of the sand and the reduction is limited to the top third of the pile. For an isolated pile, the lateral resistance reduces to approximately 40% of its static value. The author states that further testing is required to establish a comprehensive set of reduction parameters.

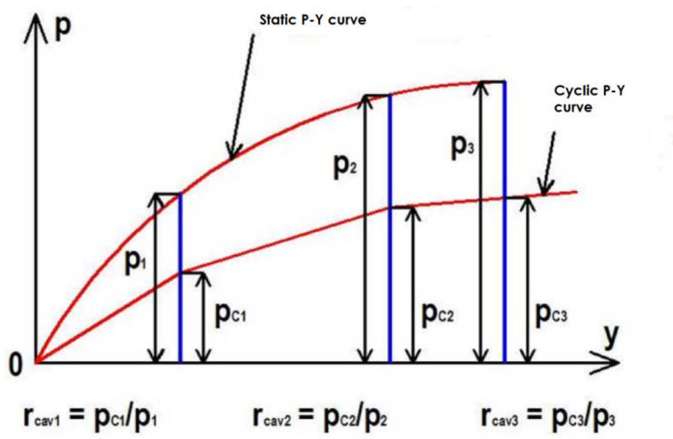

Figure 4. P-Y construction according to [7]

Dührkop [9] modifies the standard API P-Y construction based on calibration of his 1-g model tests. A conservative P-Y construction which can be used in the absence of model tests is proposed:

$$A_{cyc}(z) = 0.343\frac{z}{D} \text{ if } \frac{z}{D} \leq 0.9$$
$$p(z) = A_{cyc}(z) \cdot p_u(z) \tanh\left(\frac{k \cdot z}{0.9 \cdot p_u(z)} y\right) \tag{4}$$

The proposed P-Y model limits the lateral resistance reductions to the zone which undergoes the largest deformations. This is in accordance with the physical behavior observed in the model tests.

Although the two methods outlined above provide a useful insight into the possible reductions of lateral stiffness and lateral resistance, they are developed for monopiles under 1-way loading. Therefore, the results obtained using these methods are likely to be very conservative. Centrifuge or 1-g model testing to investigate the P-Y response of tripod or jacket piles under cyclic loading would be very useful.

In the absence of such tests, it is advisable to perform calculations for a couple of the monopile models. Even though results may differ significantly from reality, the methods can give an indication of the sensitivity of the soil surrounding the pile to the applied cyclic loading.

Recent experience on foundations in the German North Sea has shown that since the cyclic lateral loads are relatively small compared to monopiles, most of the reduced P-Y methods will give acceptable results. This indicates that significant loss of lateral capacity and stiffness is not likely to be an issue for tripod piles in the German North Sea.

3.2 Influence on axial pile resistance

As cyclic lateral loads can cause densification of the soil around the pile and a reduction of the radial stress, axial pile resistance can be affected by lateral cycling. This effect has not yet been assessed thoroughly through field or scale model tests.

For offshore pile design, a rational approach was followed whereby the unit skin friction of the pile was degraded linearly from mudline to three pile diameters below the scoured seabed. At the scoured seabed, the skin friction was fully reduced.

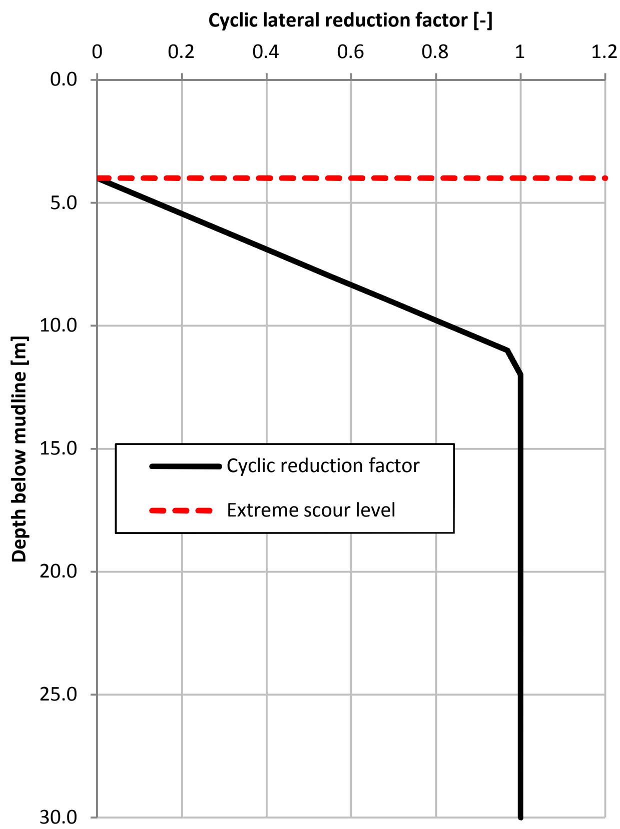

Figure 5. Reduction factor on unit skin friction for a 2.48m diameter pile (scour depth = 4m)

The reduction on the unit skin friction is applied before carrying out any axial resistance analysis.

4 INFLUENCE OF CYCLIC AXIAL LOADS ON AXIAL PILE RESISTANCE

4.1 Interaction diagrams

Long-duration combinations of average and cyclic axial loads (as occurs for tripod pile foundations) can result in a lower pile resistance than assessed for static loading [13]. Poulos proposed a cyclic stability concept categorizing the pile behavior into three zones as shown in Figure 6. The X-axis represents the ratio of the average load, E_{avg} to the characteristic pile resistance, R_k. The Y-axis shows the ratio of the cyclic load amplitude, E_{cyc}, to the characteristic pile resistance.

Poulos developed this diagram for long offshore piles in clay soils which experience degradation of the shaft resistance with displacement, and in which the axial pile flexibility contributes to higher cyclic displacements at the top of the pile than the bottom.

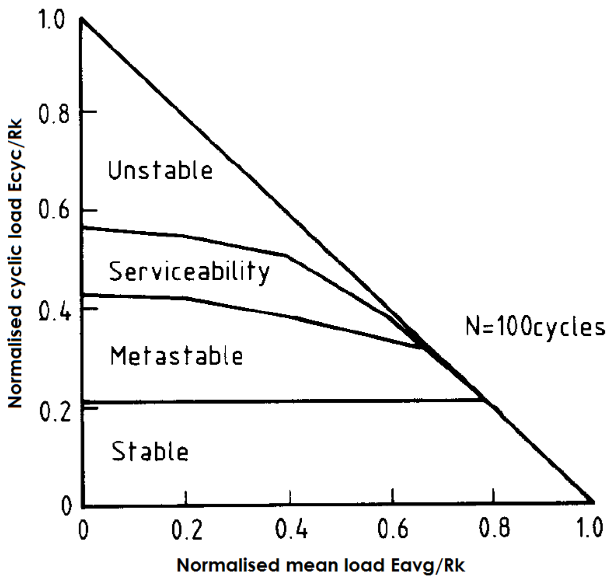

Figure 6. Cyclic stability diagram proposed in [13]

Tripod piles in dense to very dense North Sea sands are generally shorter than piles installed in clay. The cyclic axial degradation arises from densification of the sand and a subsequent reduction of the radial stress acting on the pile wall.

Based on a series of full-scale cyclic tests on driven piles in dense sand at Dunkirk [10], Jardine et al propose the interaction diagram shown in Figure 7. The tests were mainly cyclic tension tests with up to 345 load cycles. The average and cyclic load level should be normalized by the shaft capacity since the shaft resistance will be fully mobilized before significant base resistance is developed. Furthermore, Jardine et al demonstrate a significant increase of axial resistance over time. This ageing effect is not taken into account in the interaction diagram.

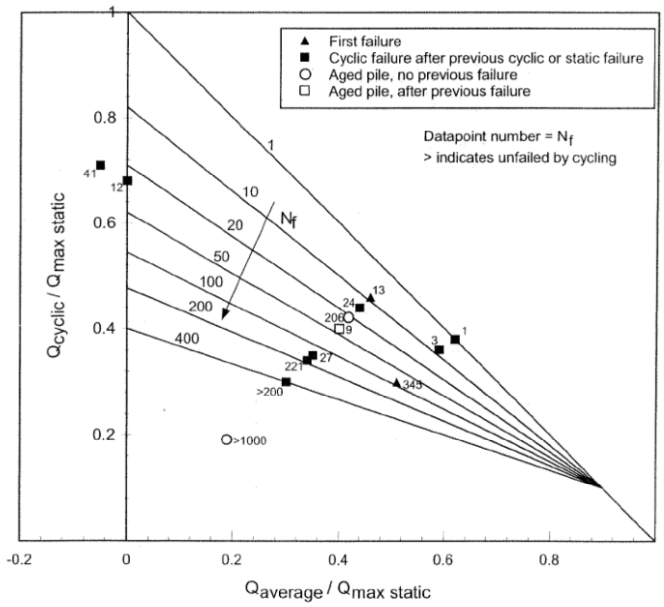

Figure 7. Cyclic interaction diagram proposed in [10]

Mittag et al [14] provide an interaction diagram up to N=1e6. The diagram was developed based on the diagram proposed by Poulos [13] extended with research results long-duration cyclic testing on small diameter piles. A distinction between soil types is not made in the diagram. The average and cyclic load level are normalized by the characteristic pile resistance in compression or tension, depending on the load regime. For piles in compression, the base resistance is included in the normalization.

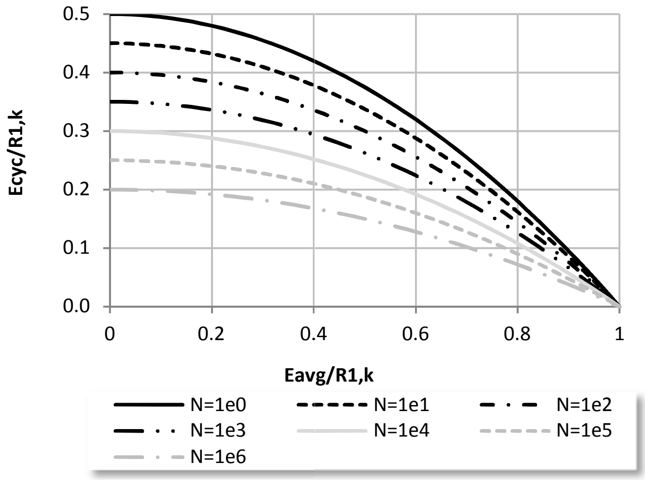

Figure 8. Interaction diagram proposed in [14]

4.2 Application of interaction diagrams to large diameter tripod piles

4.2.1 Limitations of the interaction diagrams

The three interaction diagrams proposed in this section provide a useful insight into the behavior of the pile subject to different combinations of cyclic loading. However, a number of questions remain:

- How should different combinations of cyclic and average load be combined?
- How should the average and cyclic loads be normalized (with or without base resistance)?
- Are the results for small diameter piles applicable to large offshore piles for tripod foundations?
- Is there a cut-off load level below which cyclic loading does not cause damage or even improves the pile resistance?

Although the interaction diagram in [10] is clearly the most applicable to large diameter offshore piles in dense sand which generate the majority of their capacity from skin friction, it does not provide any guidance for N>400 nor does it define a cyclic threshold below which the axial resistance is not affected. Further research on this topic is clearly required.

4.2.2 Use of interaction diagrams for irregular storm loading

According to Norsok [15], the characteristic pile resistance should be sufficient to withstand a 35hr design storm. A storm build-up chart is defined as shown in Figure 9. During the storm, the significant wave height, H_s, builds up from 50% to 100% of its maximum value. After the peak period of the storm, the significant wave heights reduce back to 50% of the maximum.

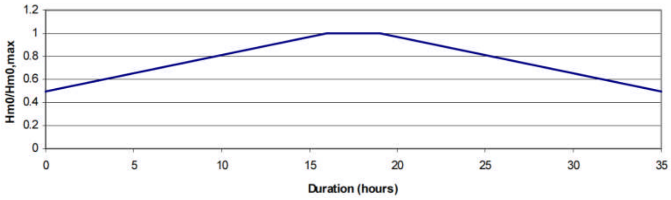

Figure 9. Storm build-up defined in [15]

Generating a storm load history based on [15] leads to a very large combination of average and cyclic load levels. Interaction diagrams were essentially developed for uniform load cycles with constant average load and cyclic load amplitude.

There are several concepts available in the literature [16][17] to bridge the gap between irregular load histories and calculation models developed for uniform cycles. The procedures outlined are developed for liquefaction analysis and soil dynamics. The application to axial pile loading is not straightforward as shown below.

The cyclic axial loads on the pile head are routinely defined according to an integral load calculation according to GL guidelines [18]. For determination of the 50-yr ULS load, a combination of the maximum 50-yr linear wave and the maximum 50-yr gust needs to be considered. This combination of wind and wave generates a peak load which can be up to 50% higher than the other loads occurring during the peak of the storm. An example of such a time history is shown in Figure 10.

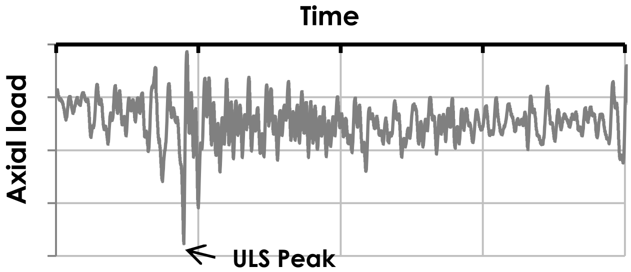

Figure 10. Example time history of axial load containing ULS event

Applying the equivalent uniform cycle concept to this load history (including the ULS peak) yields the equivalent number of cycles shown in Figure 11. The fit is very poor due to the large difference between the peak load and the other loads in the time history.

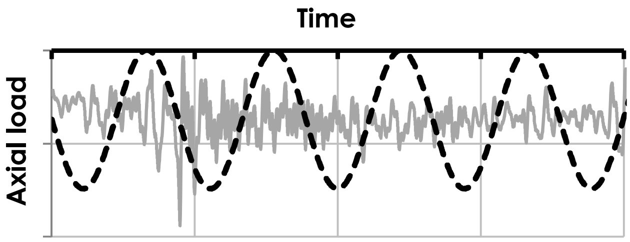

Figure 11. Equivalent uniform cycles (dashed line) for load history containing ULS event according to [16]

If the ULS event is not considered in the equivalent uniform cycle procedure, the result from Figure 12 is obtained. This fit to the other loads in the history is clearly much better.

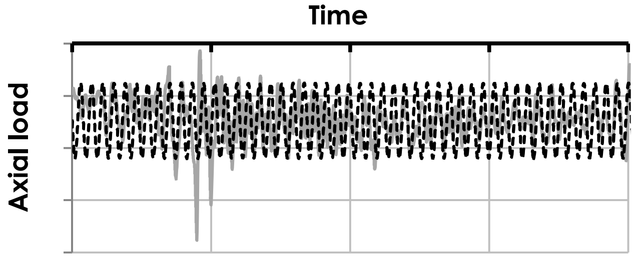

Figure 12. Equivalent uniform cycles (dashed line) for load history without ULS peak according to [16]

The equivalent uniform cycle concept clearly works much better when the peak ULS load is not considered. However, this peak load can cause a significant amount of axial resistance degradation and therefore needs to be considered in the analysis. Therefore, the application of the cyclic interaction diagrams in combination with the equivalent uniform cycle concept does not give conclusive results. A method that is capable of combining the effects of all cyclic load combinations is required for design.

4.3 Degradation analysis according based on dynamic soil properties

Richter et al [11] present a comprehensive methodology which calculates degradation of the unit skin friction based on cyclic soil properties determined from a combination of cyclic triaxial tests, cyclic DSS tests, bender element tests, resonant column tests, etc. The methodology was applied to the available database of test results showing good agreement with the measured data for both field and model tests.

Although the method is demonstrated to work well, there is only a limited amount of cyclic laboratory test data on dense to very dense sand available in the literature. Due to the large scale of most windfarm projects (typically 80 turbines per windfarm) cyclic laboratory testing at every location is not feasible due to cost and time considerations. Cyclic laboratory tests could be carried out in selected soil units after which results could be conservatively extrapolated to other locations.

4.4 Cyclic T-Z degradation analysis

The response of piles to axial loading is routinely calculated using elasto-plastic T-Z springs according to API [12]. The ultimate skin frictions calculated according to API recommendation show large statistical variations compared to field tests. In recent years, ultimate skin frictions are calculated using CPT-based methods which have been shown to be more reliable with a narrower statistical spread [19].

Based on the ultimate unit skin friction calculated using the ICP method [10], HSE developed a cyclic degradation method for use with hysteretic T-Z springs. The method is illustrated schematically in Figure 13. Due to the hysteresis of the T-Z springs, irreversible axial displacements can be accumulated during a storm simulation.

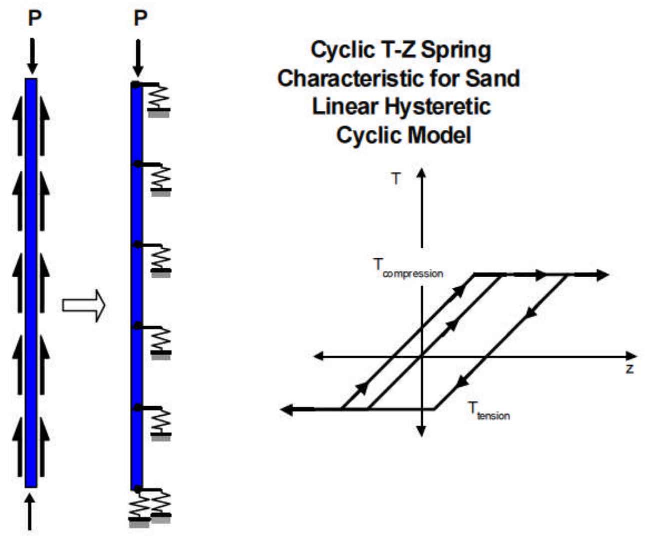

Figure 13. Hysteretic cyclic T-Z model [20]

The storm history is subdivided into blocks of cycles of constant average and cyclic load level and a T-Z analysis is computed for each cycle block. The radial stress on the pile wall is reduced based on the calculated shear stress amplitude:

$$\frac{\Delta\sigma'_{r,cyc}}{\sigma'_{ro}} = A\left(\frac{\tau_{cyc}}{\tau_{\max,stat}} + B\right)N^{C} \tag{5}$$

Where:

- $\Delta\sigma'_{r,cyc}$ is the cyclic reduction in radial stress
- σ'_{ro} is the initial radial stress calculated using the ICP method
- A, B and C are calibration parameters
- τ_{cyc} is the cyclic shear stress amplitude
- $\tau_{\max,stat}$ is the static shaft friction calculated using the ICP method and reduced for the effect of cyclic lateral loading as shown in Figure 5.

After each step, the value of $\tau_{\max,stat}$ is updated in both compression and tension:

$$\begin{aligned}\tau_{\max,stat,COMP} &= \left(\sigma'_{ro} + \Delta\sigma'_{r,cyc} + \Delta\sigma'_{rd}\right)\tan\delta \\ \tau_{\max,stat,TENS} &= 0.9\left(0.8\left(\sigma'_{ro} + \Delta\sigma'_{r,cyc}\right) + \Delta\sigma'_{rd}\right)\tan\delta\end{aligned} \tag{6}$$

Where $\Delta\sigma'_{rd}$ is the dilatant shear stress increase calculated according the ICP procedure [10].

The calibration of the parameters A,B and C was originally performed based on cyclic laboratory tests on dense sand. However, calibration to the Dunkirk pile load tests showed that the laboratory values needed to be modified. The following calibration parameters are suggested for analysis:

- A = -0.1245
- B = -0.25
- C = 0.355

The cyclic T-Z model does not require an extensive laboratory test program and it provides a rational way of combining load cycles of different average load level and amplitude.

The model has been applied to a number of North Sea oil and gas platforms [21].

The main shortcoming of the method is that it was calibrated based on the Dunkirk pile load tests which were tested up to N=345. The effects for large numbers of cycles of small to moderate amplitude cyclic loading are defined through the cut-off parameter B. However, results are sensitive to the selection of this parameter and therefore further research into the cyclic cut-off is necessary. Also, Jardine and Chow [21] mention the beneficial effects of low amplitude cyclic loading. This may accelerate the ageing process but is not taken into account in any method.

The cyclic T-Z procedure was applied to the tripod piles for a large German windfarm project to determine the required pile penetrations.

5 DESIGN APPROACH

Piled tripods are currently being planned for a number of offshore windfarms in the German sector of the North Sea. In order to achieve a safe and cost-effective foundation design in accordance with the requirements from regulating authorities [22], the different aspects of pile design outlined in this paper need to be considered.

5.1 Lateral pile design

Even though lateral loads will not be governing for pile design, the conservative P-Y construction proposed by Dührkop [9] was used to check the lateral response and the upper bound for cyclic lateral displacements. Even though this method for monopiles is very conservative, acceptable pile response was obtained.

A lower limit was imposed on the pile penetration to ensure flexible pile behavior.

5.2 Axial pile design

Axial pile resistance and accumulated axial pile head displacements are the main design drivers for tripod piles. Due to the large cyclic loads, significant degradation of axial pile resistance is possible, depending on the soil profile.

In this paper, the difficulties of applying an equivalent uniform cycle concept to storm containing a ULS event according to GL [18] are demonstrated. Interaction diagrams were used to get a general impression of the imposed axial loading but not to compute the degradation of pile resistance.

Due to its simplicity and the fact that it was calibrated to the only available cyclic axial field test program on piles in dense sand, the HSE method [20] was adopted. Analyses for the full storm were automated using the cyclic TZ algorithm CATZ, developed by Cathie Associates. The effect of cyclic lateral loading was taken into account using the reduction factor on the unit skin friction shown in Figure 5.

Results of CATZ analyses showed a degradation from 5% to 25% during the 50yr storm depending on the soil profile. An example result for a dense sand location with a 2m thick silty sand layer at 15m is shown in Figure 14. An initial reduction of 5% due to the lateral reduction factor from Figure 5 is applied at the start. During the build-up of the storm, the degradation increases progressively as the storm intensifies. The ULS event occurring around 1.5hr (at the end of the peak period of the storm causes the highest degradation. During the dissipation of the storm, the further degradation is limited.

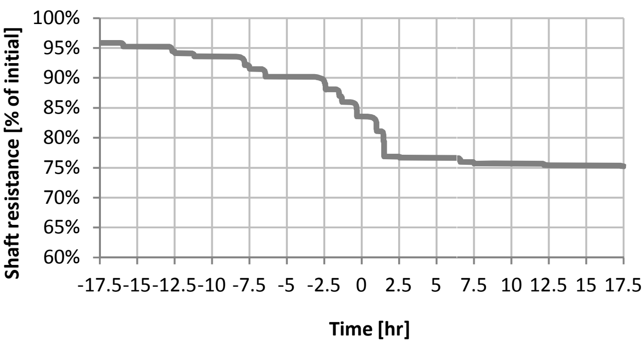

Figure 14. Axial resistance degradation during 35hr storm (dense sand profile with silty sand layer between 15m and 17m)

The degradation of the ultimate shaft shear stress is shown in Figure 15. The results show that the loads are transferred to the more competent, deeper layers as load cycling progresses. The capacity in the upper layer is significantly reduced from its initial values due to the very low residual unit shaft resistance that has been used in this example.

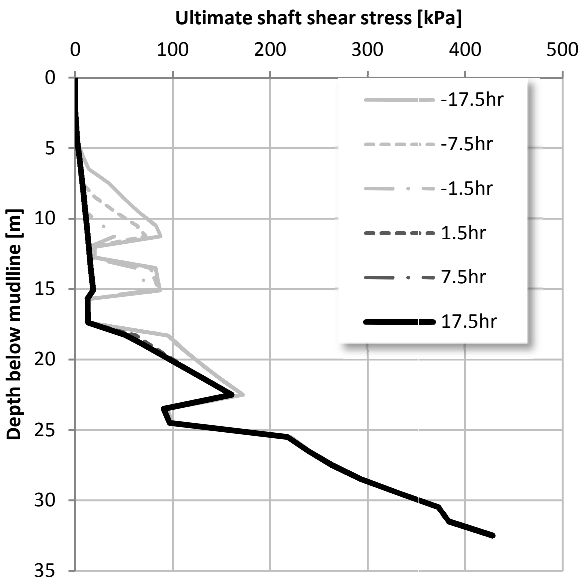

Figure 15. Degradation of shaft shear stress during 35hr storm

The degraded pile resistance is compared to the static pile resistance using the design guideline from Richter et al [11]. This leads to an increase in pile penetration between 5% and 15% compared to the pile penetration required to achieve the static resistance. The required increase in pile penetration is heavily dependent on the relative density of the sand.

6 CONCLUSIONS

This paper presents a design methodology for tripod piles undergoing cyclic axial and lateral loading. The available calculation methods for axial and lateral response are reviewed and a rational selection is made based on oil and gas experience and a critical review of the available methods.

For lateral pile response, the lack of a calculation method applicable to tripod or jacket piles is highlighted. Methods developed for monopiles can be applied and although acceptable results are obtained, they are likely to be very conservative due to the different pile response, load magnitude and load orientation.

For axial pile resistance, the CPT-based ICP method was adopted [10] in combination with a cyclic T-Z approach [20] to calculated resistance degradation during a storm. The method is straightforward to apply but the applicability to large numbers of load cycles of low to moderate amplitude needs to be investigated further.

REFERENCES

[1] M. Achmus, K. Abdel-Rahman, Y.-S. Kuo. *Behavior of large diameter monopoles under cyclic horizontal loading.* 12th International Colloquium of Structural and Geotechnical Engineering. 2007.

[2] M. Achmus, K. Abdel-Rahman, K. Thieken. *Numerical study of the effect of combined loading on the behavior of piles in sand.* International Symposium on Computational Geomechanics. Juan-les-Pins, France. 2009.

[3] N.H. Levy, I. Einav, T. Hull. *Cyclic shakedown of piles subjected to two-dimensional lateral loading.* Int. J. for Numerical and Analytical Methods in Geomechanics. DOI:10.1002. 2009.

[4] Kishida, H. Suzuki, Y., Nakai, S. (1985). Behavior of a pile under horizontal cyclic loading. 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Balkema. pp. 1413-1416.

[5] J.H. Long, G. Vanneste. *Effects of cyclic lateral loads on piles in sand.* Journal of Geotechnical Engineering. Vol. 120. No. 1. January 1994.

[6] Rosquet, F. (2004). Pieux sous charge laterale cyclique. Thèse de Doctorat. Laboratoire Central des Ponts et Chaussées de Nantes. 22/10/2004.

[7] Rakotonindriana, M.H.J. (2009). Comportement des pieux et des groups de pieux sous chargement lateral cyclique. Thèse de Doctorat. ENPC, France.

[8] C. Leblanc, B.W. Byrne, G.T. Houlsby. *Response of stiff piles to random two-way lateral loading.* Géotechnique. 26 April 2010.

[9] J. Dührkop. *Zum Einfluss van Aufweitungen und zyklischen Lasten auf das Verformungsverhalten lateral beanspruchter Pfäle in Sand.* In Technische Universität Hamburg Harburg, Veröffentlichungen des Instituts für Geotechnik und Baubetrieb, Heft 20, Hamburg. 2010.

[10] R. Jardine, F. Chow, F. Overy, J. Standing. *ICP design methods for driven piles in sands and clays.* Thomas Telford, ISBN 0 7277 3272 2, pp. 105. 2005.

[11] Richter, T., Kirsch, F., Mittag, J. (2010). Bemessungskonzepte für axial-zyklisch belastete Pfhäle – Ein Überblick und neue Ansätze. DGGT (Hrsg.) Vorträge zur Baugrundtagung 2010, München.

[12] API RP 2A-LRFD. *Recommended practice for planning, designing and constructing fixed offshore platforms – Load and resistance factor design.* First edition. July 1993.

[13] H.G. Poulos, T.S. Hull. *Pile behaviour – theory and application.* Géotechnique 39, No. 3, 365-415. 1989.

[14] J. Mittag, T. Richter. *Beitrag zur Bemessung von vertical zyklisch belasteten Pfählen.* In Festschrift zum 60. Geburtstag von Heern Prof. H.G. Kempfert. 2005.

[15] Norsok. *Actions and Action Effect.* Norsok Standard N-003. Rev. 1, February 1999.

[16] Seed H.B. *Representation of irregular stress-time histories by equivalent uniform stress series in liquefaction analysis.* Report No EERC 75-29. 1975.

[17] M. Annaki, K.L. Lee. *Equivalent uniform cycle concepts for soil dynamics.* J. Geotechnical Engineering Division, ASCE. Vol. 103, GT6, pp549-564. June 1975.

[18] Germanisher Lloyd. *Guideline for the Certification of Offshore Wind Turbines.* Edition 2005.

[19] Lehane, B.M., Schneider, J.A., Xu, K. *A review of design methods for offshore driven piles in siliceous sand.* UWA, GEO 05358, pp. 105. 2005.

[20] HSE. *Cyclic degradation of offshore piles.* Prepared by WS Atkins Consultants Ltd. Offshore Technology Report 2000/013. ISBN 0 7176 1911 7. 2000.

[21] Jardine, R.J., Chow, F. *Some recent developments in offshore pile design.* Proc. 6th Int. Offshore Site Investigation and Geotechnics conference. 11-13 September. London. 2007.

[22] BSH. *Standard design of offshore wind turbines.* 2007.